ISSN 1822-427X print
ISSN 1822-4288 online

THE BALTIC JOURNAL
OF ROAD AND BRIDGE ENGINEERING

http://www.bjrbe.vgtu.lt

2007, Vol II, No 4, 173–181

DETERMINATION OF RATIONAL PARAMETERS FOR THE ADVANCED STRUCTURE OF A PEDESTRIAN SUSPENSION STEEL BRIDGE

Algirdas Juozapaitis[1], Arnoldas Norkus[2]

[1] Dept of Bridges and Special Structures, Vilnius Gediminas Technical University, Saulėtekio al. 11, LT-10223 Vilnius, Lithuania. E-mail: alg@st.vgtu.lt
[2] Dept of Geotechnical Engineering, Vilnius Gediminas Technical University, Saulėtekio al. 11, LT-10223 Vilnius, Lithuania. E-mail: arnoldas.norkus@st.vgtu.lt

Abstract. High strength cables and steel plates or prestressed rc members (also named as stress-ribbons) usually serve the main load carrying elements of up-to-date pedestrian bridge structures. An application of these elements is prescribed actually by large magnitude of permanent load. However, a realisation of such structures requires many material resources. This investigation presents an advanced structure type for pedestrian suspension bridge created from hot-rolled cable or welded members of finite flexural stiffness. Development of displacements in such structure subjected by symmetric and asymmetric loadings is analysed. A method of stabilising displacements via flexural stiffness variation and its efficiency is considered. Displacement variation and strength of advanced load carrying structure of structure are investigated, the developed analytical expressions for determining inner forces and displacements are presented. An analysis of rational parameters for advanced structure of pedestrian suspension bridge yields expressions for determining the necessary flexural stiffness, cross-sectional height and area of load carrying structural elements. A rational primary shape of structure versus ratio of permanent and variable loadings is analysed. A technical-economic efficiency is illustrated via numerical simulation of rational parameters for advanced structure of pedestrian bridge.

Keywords: pedestrian suspension bridge, cable structure, flexural stiffness, non-linear analysis, symmetric and asymmetric loadings, rational parameters, technical-economic efficiency.

1. Introduction

Suspension structures are widely employed as load carrying structures for various types of buildings [1–7]. This feature is prescribed by technical efficiency and wonderful architectural shape of structural form. The largest spans in the world are covered by employing ability of structure to carry tensile stresses in the most efficient way [8–13]. From ancient times the suspension structures are employed for pedestrian bridges. Stress ribbon suspension pedestrian bridges distinguish amongst other ones by small height and weight [8, 14–19]. High strength steels cables or steel sheets serve as main load carrying member in such type up-to-date bridges [14, 15, 17]. Large shape changes (displacements) caused by asymmetric and/or concentrated loads is the main disadvantage of a suspension structure. The massive, reinforced concrete (rc) most often erected decks are aimed to stabilise primary form of suspension structure [8, 14, 15]. Prestressing of such rc structures every so often is applied [16–19]. Relatively large tensile inner forces develop in suspension structures due to relatively small sag and large permanent loads. This feature prescribes the large cross-sectional areas of load carrying members and the massive anchor foundations. The rigid or close to rigid support causes an additional stressing [15, 18]. Prestressed elements of such bridges are used to carry loadings both by tension and bending. Therefore one must exactly evaluate stress and strain state of such members [19]. One must note that despite the efficiency to resist static and dynamic loadings [16–21], an application of rc elements for bridges is accompanied by certain maintenance peculiarities [22–24].

Steel suspension (as well as rc) load carrying structures, erected mostly from steel sheets are employed in up-to-date engineering [8, 14]. A steel sheet as well as a cable cannot be treated as absolutely flexural cable suspension element as it has a certain height. One can state that relatively large residual bending moments develop in these elements [11, 25, 26]. Thus one can employ the flexural stiffness to monitor cable deformability. One can list the engineering solutions for roofs where the so-called "rigid" elements are employed to reduce displacements caused by asymmetric and local [5, 6, 27]. Such tensile-flexural members stabilize the primary form of the whole structure in an efficient way. Prestressing is not necessary for such structures. The structures are produced from usual rolled or welded profiles, that actually simplifies the production and erection of the load-carrying structure [5, 27]. It is important to note that the lightweight steel sheets [1, 28, 29] or efficient composite elements for flooring can be used instead of usual heavy (massive) elements [14, 30].

The "rigid" members now start to be employed versus usual flexible cables aiming to reduce structural shape changes [31–33]. In listed investigations the suspension (produced of "rigid" members) bridge behaviour is analysed, economic efficiency was proved. Technical-economic efficiency is rather important point in valuation of steel structures for bridges [34]. It is obvious that the best result can be obtained when employing optimisation methods, accounting strength, stiffness and stability constraints and nonlinear behaviour [35, 36]. The significant effect can also be achieved when composing rational cross-sectional parameters due to strength and stiffness conditions [37]. One can note that choosing a certain steel grade is amongst the definitely important features (especially for tensile members) in the considered case [38, 39].

An advanced load carrying structure of pedestrian suspension steel bridge is considered in this investigation. The usual flexible cables are replaced by the hot rolled or welded profiles, possessing the finite flexural stiffness. The reasons of developing displacements (in case of symmetric and asymmetric loadings) of suspension bridges, constructed from such elements, are discussed. Stabilisation of displacements via flexural stiffness is considered. Behaviour of advanced load carrying structure is analysed, analytical expressions for determining inner forces and displacements for "rigid" members are presented. A method for identifying the considered structure rational parameters is presented. Method yields expressions for flexural stiffness magnitude, cross-sectional height and area. Numerical simulations were performed to prove the technical-economic efficiency of rationally designed advanced structure for a pedestrian bridge.

2. Stabilisation of displacements of suspension bridge

2.1. Kinematic displacements of suspension bridge

The main load carrying element of the suspension pedestrian bridge is flexible suspension cable. Stiffness conditions in design of such a structure are the governing ones. In terms of displacements they read: $\Delta f \le \Delta f_{\lim}$, $w_{\max} \le w_{\lim}$. Suspension cable keeps the primary shape when loaded by a complement symmetric load of constant intensity p^* distributed along the whole span. The maximum vertical displacement Δf, located at middle span is prescribed only by elastic deformation [1, 5, 11]. An approximate magnitude of displacement is defined by:

$$\Delta f \cong \frac{3}{128} \frac{(q+p^*)\cdot l^4}{E\cdot A\cdot f_0^2}, \tag{1}$$

where f_0 – cable primary sag at the middle span, q – permanent symmetric load of constant intensity distributed along the whole span.

The maximal distribution to total displacement magnitude is that of kinematic displacements. The nature of these displacements is prescribed by adaptation of cable to carry loads mainly via tension. The asymmetric complement load, subjected onto one cable middle span (the most dangerous loading case), change the primary shape as shown in Fig 1. One can obtain pure kinematic displacements by introducing an infinitely large axial stiffness of cable, ie

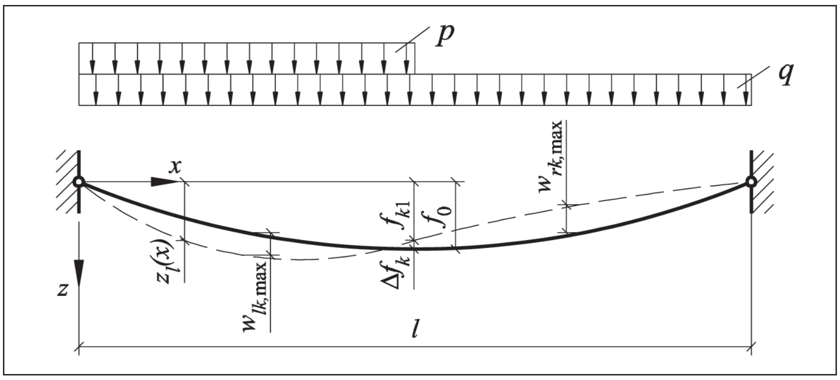

Fig 1. Change of asymmetrically loaded cable shape

$EA \to \infty$. The extreme displacements (in different directions) then develop in both middle spans of the cable.

Let's consider the case when cable is loaded by the following distributed loads: the symmetric load q per total span l and the asymmetric (supplement) load p of constant intensity per a half-span (Fig 1).

It is obvious that the kinematic sag at middle span f_{k_1} is less than the primary sag f_0 [40]. Thus the kinematic displacement at middle span is negative (ie directed up of lifted in respect of the primary position) [40, 41]. It is important to identify extreme (maximum) displacements, as it was mentioned above. For cable left part, ie loaded by a asymmetric load p the maximum vertical displacement for cable part $(x \le l/2)$ is defined by [40]:

$$\omega_{lk,\max} = f_0\left[(2\beta-\beta^2)\left(\frac{1}{\xi}-1\right)+\frac{\gamma}{2\xi}(3\beta-2\beta^2)\right],$$

$$\text{where } \beta=\frac{1-\xi+3\gamma/4}{1-\xi+\gamma}. \quad (2)$$

Taking $x^* = l/4$ from (2) one obtains on approximate formula for loaded part displacement evaluation [40]:

$$\omega_{lk,\max}(x)=\frac{3}{4}f_0\left[\frac{(1+2\gamma/3)}{\xi}-1\right]. \quad (3)$$

The above formula is rather compact and does not require complicated and large calculations. An analysis of the formula (3) proves that it produces insignificant errors when comparing with an exact solution (not exceeding 1,6 %, when $\gamma = 10$ [40, 41]).

Maximum kinematic displacement of the right unloaded cable part can be evaluated via an approximate formula [39]:

$$\omega_{rk,\max}(x)=\frac{3}{4}f_0\left[\left(\frac{1}{\xi}-1\right)+\frac{\gamma}{3\xi}\right]. \quad (4)$$

An analysis of formulae (3) and (4) show that the right cable part (free from load p) extreme kinematic displacements in absolute values are greater than the ones of the right part (ie $\omega_{rk,\max} > \omega_{lk,\max}$). Relative difference amongst these displacements vary within bounds 28 and 84 % [40, 41]. This result prima facie can be explained by the fact of always negative middle span displacement Δf_k. But one must keep in mind that negative (lifting) displacements from engineering point of view can be more dangerous as they reduce the primary curvature of cable causing tensile stresses the bridge deck.

One must note that vertical kinematic displacements are always accompanied by horizontal at an asymmetric loading [40, 41]. They are always directed to cable side loaded by supplement load p.

A calculation of bridge structures for asymmetric loading includes an evaluation of total (sum) displacements of load carrying cable in all stages of loading [1, 9, 11]. To obtain simplified expressions, compatible with practical design calculations, one proposes the total displacements to split into kinematic and elastic ones. At the first stage kinematic displacements are determined, at the second stage the elastic displacements are determined taking into account the changed geometry of adapted to loading cable. The total sag at the middle span f_1 can be presented as sum of kinematic f_{k_1} and elastic Δf_{el} sags [41]:

$$f_1 = f_{k_1} + \Delta f_{el}. \quad (5)$$

Total displacement at the cable middle point is presented as sum value, by analogy:

$$\Delta f = \Delta f_k + \Delta f_{el}. \quad (6)$$

One can obtain a simplified (approximate) formula for determining elastic displacement at cable middle point, following the above described evaluation techniques for symmetric loading:

$$\Delta f_{el} \approx \frac{3}{128}\frac{ql^4(1+\gamma/2)\psi}{E\cdot A\cdot f_{k1}^2},$$

$$\text{where } \psi=\frac{1+\gamma+\gamma^2/4}{1+\gamma+5\gamma^2/16}. \quad (7)$$

Formula (7) allows direct (excluding iterative calculation procedures) determining the elastic displacement in case of a known kinematic sag. This expressions is also convenient as allows an evaluation of the cable area A necessary to ensure cable stiffness conditions. One must note that formula (7) is general, ie it serves for determining of Δf_{el} for both loading cases, ie symmetric and asymmetric ones. When loading is symmetric, ie when $\gamma = 0$, formula (7) transforms to expression (1).

2.2. Stabilisation of displacements

Stabilising the symmetrically loaded cable

The most important task in the design of cable pedestrian bridge is to ensure satisfactory stiffness conditions, as it was mentioned above. Stabilisation of displacement in case of symmetric loading can be achieved by determining necessary cross-sectional area by:

$$A \cong \frac{3}{128}\frac{(q+p^*)\cdot l^4}{E\cdot f_0^2\cdot\Delta f_{\lim}}. \quad (8)$$

Analysis of formula (8) shows that cable area depends directly on total value of permanent and variable (supplement) load intensities. One can also reduce cable area by increasing primary sag and/or its admitted (limited) magnitude.

Stabilising the asymmetrically loaded cable

The maximal vertical displacements in case of asymmetric loading are conditioned by kinematic displacements. Two main approaches for reducing these displacements are applied in engineering practice, namely: 1) by increasing permanent symmetric load q; 2) by reducing the primary sag f_0. In some cases the prestressing of bridge structural members is employed [15, 16, 18] for the purpose. An employment of the all above-mentioned technical means results an increment of thrusting force H_{k_1}. This force necessitates to increase the cross-sectional areas of structural elements and create larger anchor foundations subsequently.

Define the ratio of symmetric load vs total load by $m = q/(q + p) = 1/(1 + \gamma)$. Applying this parameter one can fix variation of kinematic displacement values vs increment of permanent load magnitude [33]. Fig 2 illustrates a relative reduction of kinematic displacement vs parameter m. When increasing load ratio γ vary from 10 till 1, the parameter m changes from 0,091 to 0,50. Aiming to reduce ratio γ from 10 to 8, one must increase permanent load by 1,25 times, ie m magnitude changes from $m \approx 0{,}091$ by $m = 0{,}111$. One can find from the graph that the increment of q twice ($m = 0{,}167$) results reduction of maximal left part cable displacements only by 7,5 %, and that of right part by 15 %. Having increased the permanent load by 5 times ($m = 0{,}333$), the maximal left part displacements reduce by 26,5 %, the right part by 42,5 %. An increment of permanent load by 10 times causes the reduction of the above-mentioned cable parts displacements by 46 % and by 63 % respectively.

The increment of permanent load value causes changes of thrusting force. An increment of the permanent load twice $(m = 0{,}167)$ causes the increment of thrusting force by 14 %. An increment of permanent load by 5 times $(m = 0{,}333)$ causes the increment of thrusting force by 59 %. The permanent load increment by 10 times causes an increment of thrusting force by 134 %.

The reduction of primary sag f_0 leads to an analogous result. One must note that the relative increment of permanent load (when $m \geq 0{,}2$) causes a greater increment of thrusting force (in absolute values) when comparing with desired reduction of kinematic displacement magnitudes, ie the thrusting force increases relatively faster compared with the reduction of kinematic displacements.

One can note that the finite flexural stiffness EJ members of load carrying structure can be employed to reduce kinematic displacements of cable [5, 27, 31]. Both, axial and flexural stiffnesses of structural members combined together accordingly allow an efficient employing the attractive features of suspension cable and flexural beam: a rational resisting to primary shape changes via tensile and flexural deformations. Such structural members are produced from hot rolled or welded steel profiles. It is proved that "rigid" (of finite flexural stiffness) load carrying structural members are more efficient compared with a suspension cable in case of large asymmetric and local loadings [5, 27]. One can reduce a trusting force and mass of anchor foundations, subsequently by employing the lightweight deck structures in pedestrian cable bridges. It is obvious that stabilisation of primary form of asymmetrically loaded pedestrian bridge can be achieved only by choosing a certain flexural stiffness of load carrying structural elements. An efficiency of "rigid" suspension cables increases proportionally vs the increment of flexural stiffness EJ and vs the increment of primary sag [5]. Denote by η the ratio of absolutely flexible and "rigid" cable maximal displacements. Fig 3 represents the graph of variation of maximal kinematic displacements (parameter η vs slenderness pa-

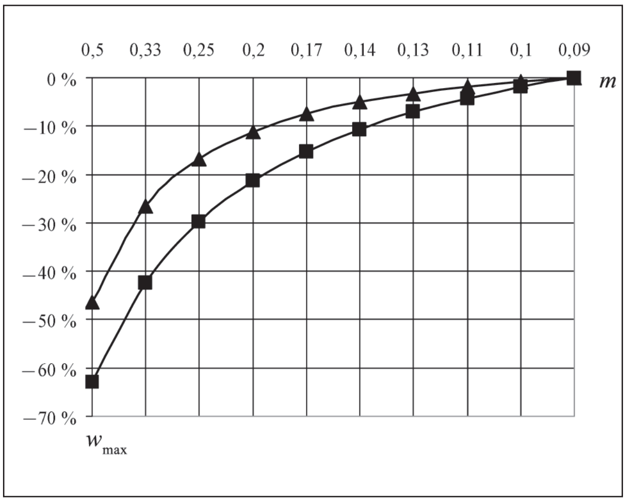

Fig 2. Left $w_{l,\max}$ and right $w_{r,\max}$ parts cable kinematic displacements relative values (%) vs parameter m (line via triangles correspond to the left cable part, line via quadrates correspond to the right cable part)

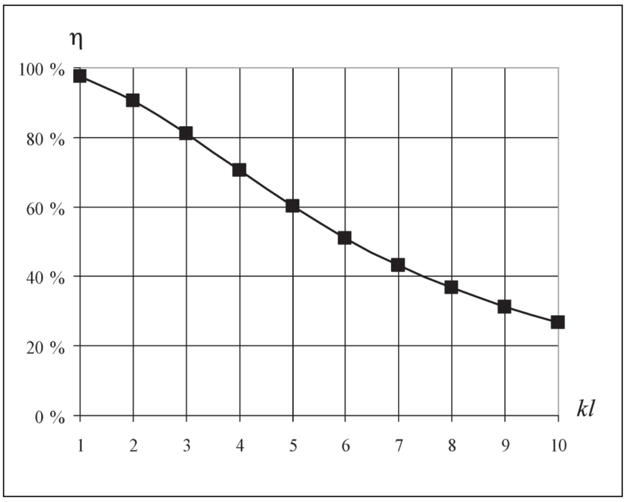

Fig 3. Difference η amongst maximal displacements of absolutely flexible and "rigid" cable vs its slenderness parameter kl

rameter $kl = l\sqrt{H / EJ}$) in case of an asymmetric loading. One can obviously find from the graph that the reduction of cable by *kl* magnitude, ie reduction of the cable flexural stiffness *EJ* causes the increment of η magnitude. For instance, for $kl = 10$ displacements of "rigid" cable are almost 27 % less, for $kl = 5$ they reduce by 60 %. In case of large flexural stiffness ($kl = 1$) cable, maximal displacements reduce approx 98 %.

An important peculiarity of stabilisation mean via variation flexural stiffness is that it does not result in a practical increment of thrusting force. Moreover, a significant increment of cable flexural stiffness reduces the thrusting force because a part of asymmetric load is carried via bending.

3. Advanced structure of a suspension pedestrian steel bridge

Main load-carrying elements of stress ribbon suspension pedestrian bridges usually are constructed from flexible cables or sheets [2–18]. Load-carrying cables or sheets are replaced by "rigid" parabola form members in investigated advanced pedestrian bridge structure. They are constructed from I type, box type or circular rolled or welded profiles. The ends of load-carrying members are connected via roller supports, to the contrary of the stress ribbon structure. This allows avoiding large support bending moments and simplifies erection works. Seeking to reduce the stresses in these "rigid" members, caused by support displacements, one can introduce the supplement third hinge at the middle span of a bridge (Fig 4). These "rigid" load-carrying elements of advanced structure avoid using technically complicated prestressing, their connections technically are simpler. Seeking to increase technical-economic efficiency of such structures, one can erect the lightweight steel or rc (under necessity) deck, and this is a technically simple work. For instance, one can apply very light thin tensile steel sheets for deck as proposed in [33]. These cylindrical form deck sheets can be distributed transversely or longitudinally in respect to load-carrying suspension structure. The supplement transverse deck beams need to be introduced for erection of longitudinal deck.

One must note that an employment of "rigid" suspension (most often the three-hinged) load carrying elements allow an efficient stabilising primary form of pedestrian bridge in case of asymmetric loadings and/or concentrated forces. One reduces thrusting forces of load carrying elements and the mass of foundations, subsequently having employed the relatively light bridge deck. Another attractive feature of "rigid" elements is that it is possible to enlarge (if maintenance conditions allow) the primary sag of load carrying structure up to 1,5–2,0 times ($f_0 \cong l/50 \div l/40$). This case does not cause large kinematic displacements but results in a significant reduction of thrusting forces. The expensive prestressed steel cables can be replaced by usual structural steel. One must note that "rigid" suspension members can be produced from straight rolled steel profiles [27]. This simplifies production of load carrying members and improves deck maintenance conditions. However, this technical solution results in larger bending moments (and subsequently stresses) when comparing with parabolic shape "rigid" cables. Thus larger member areas should be employed. On the other hand, these structural elements are more efficient in case of large asymmetric loadings. The efficiency of such structure increases if viewing-rest pitch is erected in the middle span of suspension structure [15].

4. Design of "rigid" cables of suspension bridge

Design of load carrying "rigid" element of advanced pedestrian suspension structure is performed taking into account its non-linear behaviour. Static (equilibrium), geometrical and physical equations are employed to identify inner forces and displacements. One cannot find many investigations devoted to the analysis of suspension load carrying "rigid" three-hinged [27, 33] elements. Let us present in brief the design of load-carrying structure for the considered type of bridge. Divide the structure in two inclined "rigid" cables with lower supports (central hinge can displace only vertically) for obtaining compact solutions (Fig 5). Let us create equilibrium equations in global coordinate system for separate inclined suspension members taking into account that the sag of considered pedestrian bridge is relatively small ($f_0 \cong l/60 \div l/40$) (Fig 5).

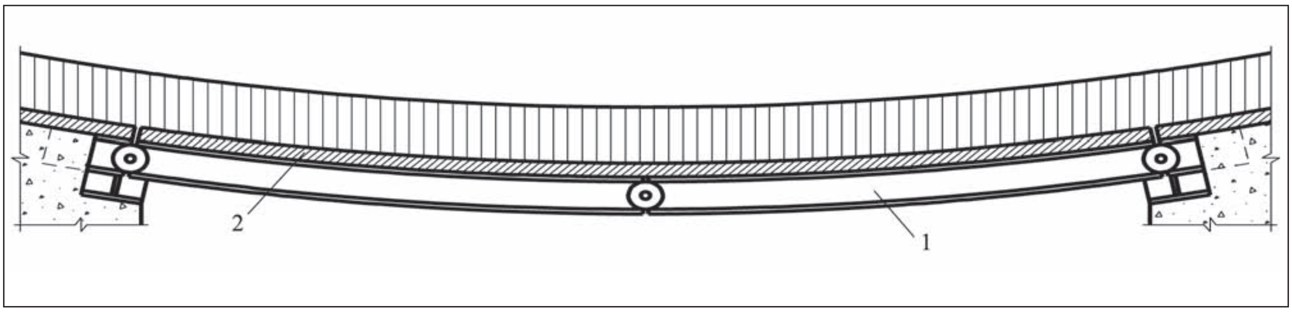

Fig 4. Advanced structure of suspension pedestrian bridge: 1 – "rigid" cable, 2 – deck structure

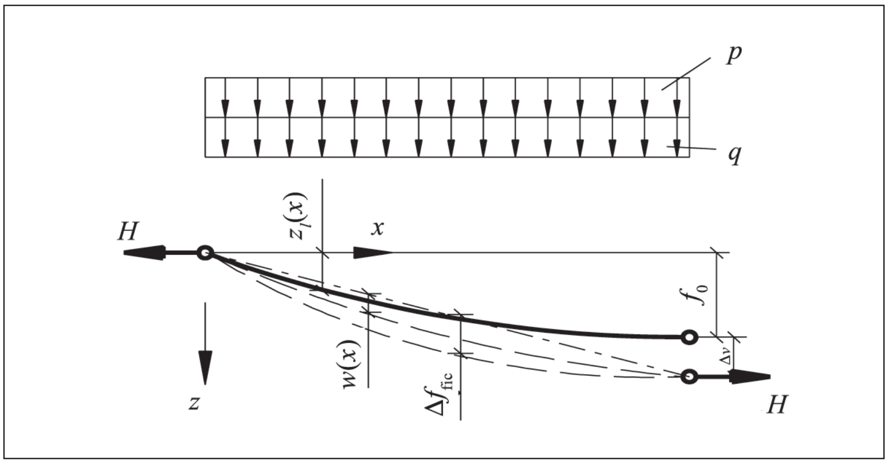

Fig 5. Design schemes for "rigid" suspension cable

An analysis of total structure (cable) yields the known thrusting force:

$$H = \frac{(q+p)L^2}{8(f_0 + \Delta v)}, \tag{9}$$

where Δv – vertical displacement of "rigid" cable at middle span (Fig 5).

One obtains the equilibrium equation for deformed state based on analysis of a separate inclined member (Fig 5):

$$EJ\, w''(x) - H[z(x) + w(x)] + \frac{(q+p)\cdot l^2}{8}\left(\frac{4x}{l} - \frac{4x^2}{l^2}\right) = 0, \tag{10}$$

where: $w(x)$, $w''(x)$ – displacement and its second derivative of inclined cable have been calculated from line connecting supports, respectively; $z(x)$ primary shape of inclined "rigid" cable (parabola).

A fictitious displacement $\Delta f_{fic,l}$ at the cable middle span is introduced aiming to reduce the number of iterative calculation procedures [5]. This allows to obtain a compact expression for determining thrusting force for left inclined suspension element. Denoting via f_{0l} the primary sag of left inclined element at middle span, one obtains:

$$H = \frac{(q+p)l^2}{8(f_{0l} + \Delta f_{fic,l})}. \tag{11}$$

From Eq (10) one obtains formula (solution) for determining displacement of "rigid" inclined element:

$$w(x) = \Delta f_{fic,l}\left[\frac{4x}{l} - \frac{4x^2}{l^2} + \frac{8}{k^2 l^2}\left(chkx + \frac{1-chkl}{shkl} shkx - 1\right)\right], \tag{12}$$

where $k = \sqrt{H/EJ}$ – slenderness parameter of inclined cable; EJ – flexural stiffness.

Bending moment of the inclined suspension element is calculated by:

$$m(x) = \Delta f_{fic,l}\frac{8EJ}{l^2}\left(\frac{chkx}{chkl/2} - 1\right). \tag{13}$$

Determining the inner forces and displacements of "rigid" cable is processed iteratively, as it was mentioned above. An elastic elongation, cable lengths prior and after deforming are employed for the purpose [5, 27]. A convergence condition for iterative calculation procedures, coupling middle cross-section (hinge) displacement of whole structure Δv with fictitious displacement of separate inclined cable part, is described by:

$$\Delta f_{fic,l} = (f_0 + \Delta v)/4\,. \tag{14}$$

Calculation principles remain the same (only computational efforts increase) when "rigid" cable is subjected to asymmetric load *p*, located at a half span of cable. Two parts of the cable are considered, namely: the left part subjected to supplement any asymmetric load *p* and the right one, subjected only to symmetric load *q*. The skew of deformed cable parts is valuated by ratio γ. The previously obtained expressions are employed for determining inner forces and displacements of both cable parts. In this case the relationship between fictitious sags (displacements) of cable left and right parts is realised by:

$$f_{fic,l} \cong f_{fic,r}(1+\gamma)\,. \tag{15}$$

Central hinge of structure is displaced in both, vertical and horizontal, directions. Convergence condition for iterative calculation procedures is realised by:

$$\Delta f_{fic,m} = (f_0 + \Delta v)\frac{(1+\gamma)}{(1+\gamma/2)\cdot 4}. \quad (16)$$

One can determine any cross-sectional displacement $w(x)$ and bending moment bending moment $m(x)$ when fictitious displacements of cable parts $\Delta f_{fic,l}$ ($\Delta f_{fic,r}$) and thrusting force H are already identified.

It necessary to note that three-hinged "rigid" cables are more efficient than the analogous two-hinged structures (taking into account the always existing horizontal displacement of cable supports).

5. Evaluation of rational parameters for "rigid" cables of pedestrian bridge

Dominating constraints in design of a suspension pedestrian bridge are the stiffness ones, as mentioned above. Elastic displacements are the governing ones in case of symmetric loading and kinematic displacements in case of an asymmetric loading. The first ones can be stabilised by increasing cable axial stiffness EA. Kinematic displacements are stabilised by choosing the necessary cable flexural stiffness EJ. Which of two, namely symmetric or asymmetric loading, becomes the governing one, depends on a set of parameters: primary sag f_0, ratio of loads γ, steel strength limit $f_{y,d}$. Flexural stiffness EJ of a "rigid" cable can be determined by employing the parameter kl into iterative calculation procedures (12) for known thrusting force H, limiting (admitted) magnitude of displacement $w_{\lim}$. For the starting point of iterative procedures the magnitude EJ can be chosen by applying an approximate formula:

$$EJ \geq \frac{5}{768}\frac{p \cdot l^4}{w_{\lim}}. \quad (17)$$

But for the known EJ one has not an idea what shape and area cross-section satisfies both the aforementioned stiffness and strength condition $\sigma_{\max} \leq f_{y,d} \cdot \gamma_c$. It is obvious that variability design (random selection) method is not sufficiently efficient as the chosen parameters prescribe the magnitudes of inner forces and displacements to develop. A minimum necessary area according the strength condition is calculated by:

$$A \geq \frac{H}{f_{y,d}}\left(1 + \frac{e}{2\cdot\alpha^2\cdot h_c}\right), \quad (18)$$

where $e = m_{\max}/H$ – eccentricity value; h_c – cable cross-sectional height; α – form coefficient of cross-section.

Formula (18) shows that the cross-sectional area depends not only on strength limit but also on its height and form. The second item of formula (18) shows an influence of cross-sectional area on the magnitude of bending moment. Prima facie it appears that an increment of height results in a smaller area. But variation of cross-sectional height causes a variation of eccentricity, which influence the magnitude of area subsequently. One can obtain the rational cross-sectional height, ensuring the minimum area by employing the slenderness parameter $kl = l\sqrt{H/EJ}$. Substituting into it the thrusting force and second moment area magnitudes and having solved the quadratic equation, one finally obtains the formula for rational cross-sectional height:

$$h_c = \sqrt{(e/4\alpha^2)^2 + f_{y,d}/k^2E\cdot\alpha^2} - (e/4\alpha^2)^2. \quad (19)$$

One can see that a cross-sectional height of "rigid" cable depends on the ratio of inner forces (eccentricity), the steel strength limit and its cross-sectional form. It is obvious that a higher strength limit prescribes the higher height of cross-section. Formulae (18)–(19) allow choosing the rational cross-sectional height and area of suspension cable for selecting its form. One must note that cross-sectional form also prescribes its area magnitude. Thus one recommends in design of structure in each case to choose the rational cross-sectional form, satisfying strength and stiffness conditions for a minimal cross-sectional area. This aim is achieved by performing the iterative calculation procedures.

Numerical simulation for choosing the rational cross-sectional area of suspension cable was performed aiming to view an efficiency of the obtained method and techniques. Analytical solution method and COSMOS M FEM software for non-linear analysis were employed for the purpose. An influence of primary sag f_0 was also analysed. Strength and stiffness conditions were verified in case of symmetric and asymmetric loadings. The 20 m span covered by a suspension structure with variation of primary sag from 0,25 till 1,0 m was considered. The rolled profiles were designed from steel S355. It was proved that a usual (variant) selection of profiles for "rigid" cable always yields larger cross-sectional area versus the one obtained via analytical relations (19, 18). The chosen cross-sectional heights were by 9–54 % smaller than the rational one. The cross-sectional areas chosen in a usual way were by 11–58 % larger than the ones, obtained via analytical formulae. One must note that rolled I type profiles violated the composition principles (prescribed by formulae (19) and (18) of rational cross-section. Welded I type profiles were applied in this case. It was proved that having reduced the primary sag twice (from 0,25 to 0,5 m), the rational cross-sectional height reduced almost by 19 % under the same admitted limit displacement magnitude ($w_{\lim} = L/400$). Cross-sectional area was reduced almost by 34 %. The latter results prove rationality of "rigid" suspension members versus the absolutely flexural ones. The larger kinematic displacements, ie when stiffness conditions have a grater influence, result the "rigid" cables to be more efficient. It was proved that one can save on average 35–160 % steel resources when employing the "rigid" cables in case when loads ratio γ and primary sag f_0 increase.

6. Concluding remarks

The proposed advanced load-carrying structure for a pedestrian bridge proved its technical-economic efficiency versus usually applied flexible suspension or rc pedestrian bridge. The proposed method and techniques for evaluating structural behaviour and choosing rational parameters were presented via design-ready relatively simple formulae, compatible with an usual conventional design. Thus they can be easily implemented into the practical design. The efficiency of proposed structural analysis/design techniques are illustrated by a numerical simulation of cable for advanced suspension steel pedestrian bridge. Reliability of obtained results is confirmed by close results obtained by a simulation of considered structure via FEM software of non-linear analysis.

References

1. KULBACH, V. *Cable structures. Design and static analysis.* Tallin: Estonian Academy Publishers, 2007. 224 p.
2. LEWIS, W. *Tension structures: form and behavior.* London: Thomas Telford Ltd., 2003, 256 p.
3. BUCHOLT, H. A. *An introduction to cable roof structures.* London: Thomas Telford Ltd., 1999. 283 p.
4. MOSKALEV, N. S.; POPOVA, R. A. *Steel structures of lightweight buildings* (Стальные конструкции легких зданий). Moscow: ACB, 2003. 215 p. (in Russian).
5. PALKOWSKI, Sz. Some problems of calculation and design of cable structures. In *Proc of the 11th International Conference on Metal Structures: Progress in Steel, Composite and Aluminium Structures*. Rzeszow, Poland, 2006, p. 102–116.
6. GOREV, V. *Steel structures. Structures of buildings* (Металлические конструкции. Конструкции зданий). Moscow: Vyschaja schkola, 2002. 527 p. (in Russian).
7. MICHAILOV, V. V. *Prestressed combined bar and cable structures* (Предварительно напряженные комбинированные и вантовые конструкции). Moscow: ACB, 2002. 256 p. (in Russian).
8. TROYANO, L. F. *Bridge engineering. A global perspective.* London: Tomas Telford Ltd., 2003. 775 p.
9. *Manual of bridges engineering.* Edited by M. J. Ryall, G. A. R. Parke and J. E. Harding. Tomas Telford Ltd., 2000. 1007 p.
10. BENNETT, D. *The architecture of bridge design*. London: Thomas Telford Ltd., 1997. 200 p.
11. GIMSING, N. J. *Cable supported bridges – concept and design*. Second edition. John Wiley & Sons, Chichester, 1997. 471 p.
12. BANGASH, M. Y. *Prototype bridge structures: analysis and design*. London: Thomas Telford Ltd., 1999. 1171 p.
13. KRISHNA, P. Tension roofs and bridges. *Journal of Constructional Steel Research*, 2001, Vol 57, No 11, p. 1123–1140.
14. SCHLAICH, J.; BERGERMAN, R. *Pedestrian bridges* (Fußgängerbrücken). Zürich (ETH): Schwabische Drückerei GmbH, 1992. 83 p. (in German)
15. STRASKY, J. *Stress-ribbon and supported cable pedestrian bridges*. London: Thomas Telford Ltd., 2005. 240 p.
16. REDFIELD, CH.; STRASKY, J. Blue valley ranch bridge. In *Proc of the 6th International Conference on Short and Medium Span Bridges.* Vancouver-Montreal, 2002, p. 1127–1134.
17. SCHLAICH, J.; SCHLAICHM, M.; WERWIGK, M. New Glacisbridge Ingolstad (Die neue Glacisbrücke Ingolstad). *Beton und Stahlbetonbau*, 1999, Vol 94, No 11, p. 466–475 (in German).
18. ATANASOVSKI, S.; MARKOVSKI, G. Design of the pedestrian bridge over river Vardan in Skopje, Republic of Macedonia. In *Cable-Supported Bridges – Challenging Technical Limits: IABSE Conference, Seoul, Korea – June 12–14, 2001 Reports*, 2001, Vol 84, p. 82–83 (+ CD).
19. ZAMBLAUSKAITĖ, R.; KAKLAUSKAS, G.; BAČINSKAS, D. Deformational analysis of prestressed high-strength concrete members using flexural constitutive model. *Journal of Civil Engineering and Management*, 2005, Vol 11, No 2, p. 145–151.
20. CAETANO, E.; CUNHA, A. Experimental and numerical assessment of the dynamic behavior of a stress-ribbon footbridge. *Structural Concrete*, 2004, Vol 5, No 1, p. 29–38.
21. PAEGLITIS, A.; SAHMENKO, G. Bridges with lightweight aggregate concrete structures. *The Baltic Journal of Road and Bridge Engineering*, 2006, Vol I, No 1, p. 55–61.
22. KAMAITIS, Z. Deterioration of bridge deck roadway members. Part II: condition evaluation. *The Baltic Journal of Road and Bridge Engineering,* 2006, Vol I, No 4, p. 185–192.
23. KAMAITIS, Z. Structural design of polymer protective coatings for reinforced concrete structures. Part I: theoretical considerations. *Journal of Civil Engineering and Management*, 2007, Vol 13, No 1, p. 11–17.
24. KAMAITIS, Z. Structural design of polymer protective coatings for reinforced concrete structures. Part II: experimental verification. *Journal of Civil Engineering and Management*, 2007, Vol 13, No 1, p. 19–26.
25. FURST, A.; MARTI, P.; GANZ, H. Bending of stay cables. *Structural Engineering International*, 2003, Vol 13, No 1, p. 42–46.
26. PRATO, C, CEBALLOS M. Dynamic bending stresses near the ends of parallel bundle stay cables. *Structural Engineering International,* Vol 13, No 1, 2003, p. 42–46.
27. JUOZAPAITIS, A.; ŠARASKIN, V.; GRIGORJEVA, T.; VALIŪNAS, B. Analysis and arrangement of suspension structures from straight-line elements of finite flexural stiffness (Расчет и компоновка висячих конструкций из прямолинейных элементов). *Theoretical Foundations of Civil Engineering. Polish-Ukrainian Transactions,* 2002, Vol II, p. 887–896 (in Russian).
28. TROFIMOV, V, KAMINSKIJ A. Light-weight metal structures of buildings (Легкие металлические конструкции зданий и сооружений). Moscow: ACB, 2002. 575 p. (in Russian).
29. JUOZAPAITIS, A.; KVEDARAS, A.K. Innovative structural system of steel roofs. *Journal of Constructional Steel Research*, 1999, No 49, p. 213–221.
30. VALIVONIS, J. Analysis of behavior of contact between the

profiled steel sheeting and the concrete. *Journal of Civil Engineering and Management*, 2006, Vol 12, No 3, p. 187–194.

31. GRIGORJEVA, T.; JUOZAPAITIS, A.; KAMAITIS, Z. Bending stiffness analysis of the main cables of cables-supported bridges. In *Proc of the 2nd International Conference of the International Association for Bridge Maintenance and Safety (IABMAS): Bridge Maintenance, Safety, Management and Cost*, Kyoto, Japan, 18–22 Oct, 2004, p. 859–860 (+CD).
32. GRIGORJEVA, T.; JUOZAPAITIS, A.; KAMAITIS, Z. Simplified engineering method of suspension bridges with rigid cables under action of symmetrical and asymmetrical loads. *The Baltic Journal of Road and Bridge Engineering,* 2006, Vol I, No 1, p. 11–20.
33. JUOZAPAITIS, A.; VAINIŪNAS, P.; KAKLAUSKAS, G. A new steel structural system of a suspension pedestrian bridge. *Journal of Constructional Steel Research*, 2006, No 62, p. 1257–1263.
34. FARKAS, J.; JARMAI; K. *Economic design of metal structures*. Rotterdam: Millpress, 2003. 339 p.
35. KARKAUSKAS, R.; NORKUS, A. Truss optimization under stiffness, stability constrains and random loading. *Mechanics Research Communications*, 2006, Vol 33, p. 177–189.
36. KARKAUSKAS, R. Optimisation of geometrically non-linear elastic-plastic structures in the state prior to plastic collapse. *Journal of Civil Engineering and Management,* 2007, Vol 13, No 3, p. 183–192.
37. JUOZAPAITIS, A.; ŠAUČIUVĖNAS, G.; NAGEVIČIUS, J. Strut-framed beam structure for reconstruction of pedestrian bridges. *Technological and Economic Development of Economy,* 2007, Vol XIII, No 2, p. 126–133.
38. *Use and application of high-performance steels for steel structures*. Structural Engineering Documents 8, IABSE, 2005. 152 p.
39. STRAUSS, A.; KALA, Z.; BERGMEISTER, K.; HOFFMAN, S.; NOVAK, D. *Technological peculiarities of steels in European view* (Technologische Eigenschaften von Stählen im europäischen Vergleich). Stahlbau, 2006, Heft 1 (in German).
40. JUOZAPAITIS, A.; NORKUS, A. Displacement analysis of asymmetrically loaded cable. *Journal of Civil Engineering and Management*, 2004, Vol 10, No 4, p. 277–284.
41. JUOZAPAITIS, A.; NORKUS, A.. Shape determining of a loaded cable via total displacements. *Technological and Economic Development of Economy,* 2005, Vol XI, No 4, p. 283–291.

Submitted 6 Sept 2007; accepted 19 Nov 2007